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Session 2: Discussion & Replies

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Discussion by Wang Zhong-qi
Deputy Chief Engineer, Academy of Building Research of China on Site Analysis for Seismic Soil Liquefaction Potential by James B. Forrest, John M. Ferritto, and George Wu.

Mr.Forrest et al have presented very interesting data showing that field sounding techniques of various kind indicated that a recently deposited silty sand layer having a liquefaction resistance approximately one half that based on cyclic triaxial testing. By my opinion, this is due to the fact that silty sand has double behaviour of both granular material and cohesive one, thus its resistance to liquefaction depends mainly upon its shear strength and percentage of clay particles which affect the activity of soil skeleton.

In view of the confusion in using criteria for liquefaction assessment, and in the light of the particular behaviour of the silty sands, lots of research works have been undertaken in China. A new approach (1) by using the electrical static cone penetrometer which was developed and propagated in our country early from 1964 (2) provides fairly good results in liquefaction prediction for silty sand with particular interest in Tienjing district.

The Chinese static cone penetrometers were made up with some special techniques to ensure highest sensitivity in transducing mechanical forces from either the cone point or the sleeve of the cone, meanwhile it has reliable watertightness without O-ring seal in order to avoid frictional error. The most commonly used probe is a so called single-bridge sonde which has been defined by a designation p_s called the specific penetration resistance. (Fig. 1)

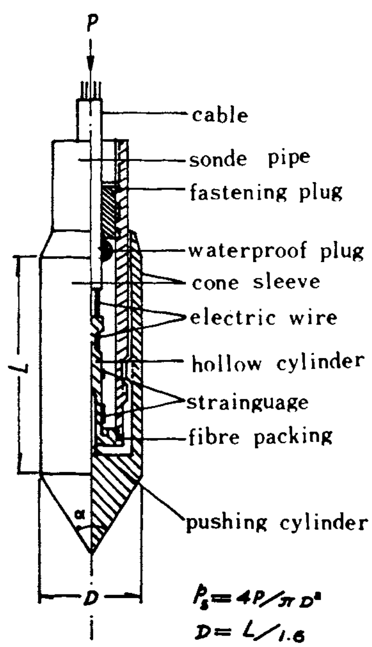

Fig. 1
Schematic set-up of one of the Chinese electrical static penetrometers — single bridge sonde.

As a result of statistically analytical study of the liquefaction in silty sands with liquefied and non-liquefied case histories, the following equation for a critical value of p_s was achieved:

$$p_{so} = 70.2 - 4.51\ M_c \qquad (1)$$

Where p_{so} is the critical value of liquefac- for p_s; M_c is the fraction in percentage of clay particles. When the measured $p_s < p_{so}$, soil is likely to liquefy; otherwise, unlikely to liquefy. This assessment is effective for a future earthquake of 8th grade of the Chinese Intensity Scale (approx. to MMS).

Taking the overburden pressure and ground water table into account, the criteria for prediction liquefaction in silty sand layers situated in seismic zone of 8th grade of the Chinese Intensity Scale is as follows:

$$Z = p_s + 117.2K - 1.318d_s + 4.316d_w - 72.27 \qquad (2)$$

where, K - ratio of clay particles to silt particles (M_c/M_s); d_s - embedded depth of silty sand layer to be studied; d_w - depth of ground water; when $Z > 0$, no liquefaction is likely to occur, $Z \leq 0$, liquefaction is likely to occur.

I suppose, this new approach in China would help in dealing with liquefaction problems in silty sand layer.

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Discussion by Wang Zhong-qi, Deputy Chief Engineer, Academy of Building Research of China on "Field Correlation of Soil Liquefaction with SPT and Grain Size by K. Tokimatsu and Y. Yoshimi.

Mr. Tokimatsu and Mr. Yoshimi have made a successful comparison between the two methods for field prediction of soil liquefaction by Dr. Seed and Iwasaki-Tatsuoka respectively. They pointed out that the Seed method tends to underestimate the resistance to liquefaction for small N-values particularly for silty sands; whereas the Iwasaki-Tatsuoka method tends to underestimates the resistance to liquefaction for large N-values. I fully agree with the author's viewpoints because I have had similar experience in China.

For last ten years, controversies over SPT and numerous comments relating its use and abuse have been seen in many papers. As for the application in soil liquefaction problems, the following key points should be put under consideration.

1. The Correction of N-value with Depth

Seed (1) proposed that the measured penetration resistance N should be corrected to an effective overburden pressure of one ton per square foot (σ_1') based on the results given by Gibbs and Holtz, i.e.

$$N_1 = N\left(1 - 1.25\log\frac{\sigma_0'}{\sigma_1'}\right)$$

where σ_0' is the corresponding effective overburden pressure in tsf. But, it should be noted that when penetration goes with increasing depth, more rods and couplings will introduce much more factors affecting N-value. In general, the absorption of impact energy transmitted through the rod system will cause the N-value to increase falsely. For such a correction, N-value should be reduced with a fraction " " shown in Fig.1 (2):

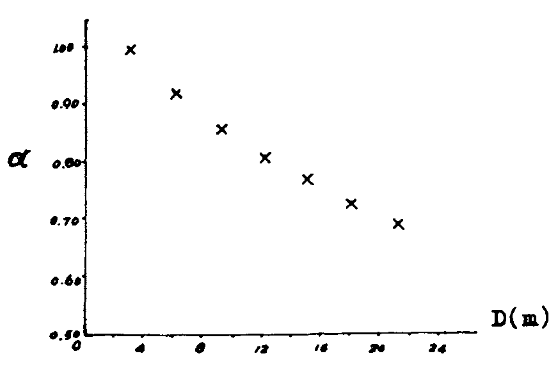

Fig.1 Depth of Penetration in Meter versus α-value

2. Measures to be taken for Eliminating Accidental Errors

Although numerous propositions have been presented to such an end so far (3), I am still convinced by the idea that the principal improvement of SPT should be laid with emphasis on the performance of SPT.

There are a great deal of sources of errors which may alter N-value of SPT. Those due to inadequate manipulation or unsatisfactory performance of the testing equipment other than that caused by non-standardized specifications are pointed out in the following ("+" means causing incorrect increase of N-value, "-" means erroneous decrease of N-value).

(1) Inadequate cleaning of the borehole. Lots of sludge may be trapped into the sampler spoon. (+)

(2) When using casing, driving sampler spoon within the bottom of the casing. ("+" for sand, "-" for cohesive soils)

(3) When using drilling mud, inadequate consistency (or specific gravity) causing wall collapse or bottom heaving of borehole. (+)

(4) The loosening of coupling of drill rod during continuous hammering. (+)

(5) Buckling of drill rod during driving. (+)

(6) Extreme length of drill rod with the increase of penetration depth causing excess absorption of impaction energy. (+)

(7) Not co-axial of connected drill rods. (+)

(8) Excess of overburden pressure combined with the increase of penetration depth. (+)

(9) Too low of the water head in borehole causing quick sand on the bottom of the hole. (+)/(-)

(10) Using solid stem auger to produce partial vacuum on the bottom, and causing failure of soil. (-)

(11) The effect of borehole diameter -- the larger the diameter the smaller the N value. (-)

(12) When using monkey rope-slip winch system to make a hammer drop, considerable friction has been exerted to diminish the driving energy. (+)

(13) When using free fall hammer the clamp of the hammer strikes back upward eventually. (+)/(-)

(14) When wash boring following the test, pumping capacity too high or too low. (-)/(+)

(15) Borehole inclined as to cause the rod stick to the hole. (+)

(16) Hammer drop inaccurate in distance due to manually controlled wire rope system. (-)/(+)

(17) Too thick and too heavy of the rod. (+)/(-)

(18) No preliminary driving before normal counting. (-)

(19) Big gravel or cobbles imped sampler spoon. (+)

(20) Liquefaction of sand layer directly overlying above the watertable due to driving. (-)

The affecting factors listed above arises in different cases. Some of them (as No.(1), (2), (3) etc.) are due to improper performance of the test, and improvements of operation are needed. Some else (as No.(4), (5) etc.) come from the deficiencies of testing equipment and could be eliminated by improving facilities. Besides, there are some original shortcomings of the SPT (such as No.(6), (8) etc.), and need some modifications to the testing data.

In view of the important role played by the SPT in assessing soil liquefaction potential, the factors listed above should be normalized and specified.

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(3) Schmertmann, J.H. (1977): "Use the SPT to measure Dynamic Soil Properties ? ----Yes. But...." proc. ASTM Symposium on Dynamic Field and Laboratory Testing of Soil and Rock Denver, Colorado

Discussion by Pedro A. DeAlba, Assistant Professor, Civil Engineering, University of New Hampshire on "Macroscopic Approach to Soil Liquefaction" by Wang Zhong-qi.

This paper presents extremely interesting field data on surface manifestations of liquefaction. These observations are for the most part from the Tangshan earthquake of July 28, 1976; this event, of magnitude 7.8, had its epicenter within the limits of Tangshan City and produced liquefaction over an area of remarkable extent: 3000 km^2 exhibited "serious sand blowing" out of a total area of 24,000 km^2 in which sand blows were observed (Chen, 1979).

The author describes two common manifestations of excess pore pressure relief following the liquefaction of a soil layer at depth: isolated sand boils and sand-filled cracks. He is correct in suggesting that geological and topographical factors determine the form of surface manifestations.

It might be expected that, in a level ground profile consisting of reasonably uniform cohesionless soil, excess pore water pressure relief would theoretically express itself by a uniform rise in the groundwater table with consequent flooding if the original groundwater level is near the surface. However, it has been repeatedly observed that, instead, pressure is relieved through the formation of a multitude of isolated spouts resulting in fields of sand volcanoes.

It is the discussor's view that the formation of sand-filled cracks, on the other hand, is indicative of a cohesive layer overlying the liquefied deposit, concentrating pressure relief along planes of weakness. This view is supported by the author's Fig. 3.4 showing surface cracking in the vicinity of river bends. This cracking pattern ("network pattern"), which tends to be parallel to the axis of symmetry of the bend, might be interpreted as reflecting the spreading tendency of the river banks towards the riverbed as underlying soil layers liquify. This same phenomenon of extensive bank movement towards the river has been reported for an earthquake in Chiapas, Mexico, by Flores-Berrones and Dawson (1977).

The tortile pattern of cracking shown in Fig. 3.6 of the paper may be a more complex manifestation of the same effect including a tendency to spread towards old, filled-in, channels discernible in the photograph.

Thus, sites in alluvial plains where liquefaction-susceptible sands are overlain by cohesive soils may suffer extensive cracking for a considerable distance away from the river banks. Structures sited in such locations may suffer heavy damage as is shown in the author's Fig. 3.5.

Another problem of particular interest to the siting of structures which is pointed out in this paper is that once a site has liquefied, it will reliquefy repeatedly in shocks of smaller intensity than the original event.

It might be suggested that reliquefaction occurs in cohesionless soils overlying the primary liquefied layer which were loosened by the upward flow of water during the original liquefaction. This loosening effect is clearly shown in Figs. 1.4 to 1.6 of this paper. These figures also show that repeated reliquefaction and reconsolidation will bring the material back towards its original density, but very slowly.

This reliquefaction model may of course be complicated by reliquefaction of the original liquefied layer; while it might be argued that, upon dissipation of excess pore pressure, the material density will increase, thereby increasing liquefaction resistance, it is disturbing to note that laboratory reliquefaction tests show that once a sample has liquefied, it may be reliquefied at lower cyclic stress levels than originally applied, in spite of being allowed to reconsolidate (Finn et al, 1970).

Thus reconstruction at a site that has liquefied should be preceded by careful site improvement measures if further liquefaction damage is to be avoided.

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Discussion by Y.P. Vaid
Associate Professor of Civil Engineering,
The University of British Columbia,
Vancouver, Canada, on Undrained Behavior
of Cohesionless Soils under Cyclic and
Transient Loading by M.P. Luong and
J.F. Sidaner.

I would like to make some observations on the paper by Luong and Sidaner. In addition, I shall illustrate that extremely careful experimentation is needed in order to measure soil properties with confidence. Reliable experimental data is a prerequisite both in the development of constitutive models of soil behaviour and in analytical solutions using soil parameters determined from laboratory tests. This question of laboratory measurement is thus addressed to those contributors of this session who make use of the laboratory test data in some form.

The Characteristic Threshold

The concept of characteristic threshold (CT) proposed by Luong and Sidaner is very valuable for a fundamental understanding of the deformation response of sand under undrained loading. Recent studies of cyclic undrained behaviour of sand at the University of British Columbia substantially support Luong and Sidaner's conclusions as to the CT line being a boundary dividing regions of contractive and dilative responses. The CT line is not only independent of the relative density of sand, as reported by the authors, but was also found independent of initial consolidation stress ratio ($\sigma'_{1c}/\sigma'_{3c}$) and cyclic stress ($\sigma_{dy}$) level.

However, one major difference we note in contrast to the authors' finding is that for loose initial densities, contractive deformation can occur for stress states lying between CT and FL lines. For such densities, the arrival of the effective stress state on the CT line triggers the onset of a large contractive deformation, which is similar to that reported by Castro (1975) in monotonic loading tests. Fig. 1 shows results from a typical cyclic loading undrained test on isotropically consolidated loose Ottawa sand. This figure shows that as soon as the effective stress state of the sample reached the CT line (point A), contractive flow deformation occurred, accompanied by a decrease in deviator stress and a sudden development of large (over 7%) axial strain. This flow deformation stopped at point B when the sample strained sufficiently so as to cause dilation with further straining (section BC on stress path). Further unloading of the shear stress to zero led to the development of a transient state of zero effective stress (liquefaction). Subsequent loading resulted in the effective stress state to move along the FL line during increasing shear stresses and development of large porewater pressure leading to liquefaction on decreasing shear stresses to zero. Repetition of this phenomena was found responsible for further development of strain with cyclic loading.

The undrained behaviour of sand at medium and dense relative densities was found essentially similar to that observed by the authors. In such cases accelerated rate of strain development occurred only after the stress state had crossed the CT line. It is important to note here that a sudden development of strain due to flow deformation is a characteristic of loose sands only. In dense and medium sand no flow deformation develops, but the strain increases steadily with cycles of loading once the stress state has crossed the CT line.

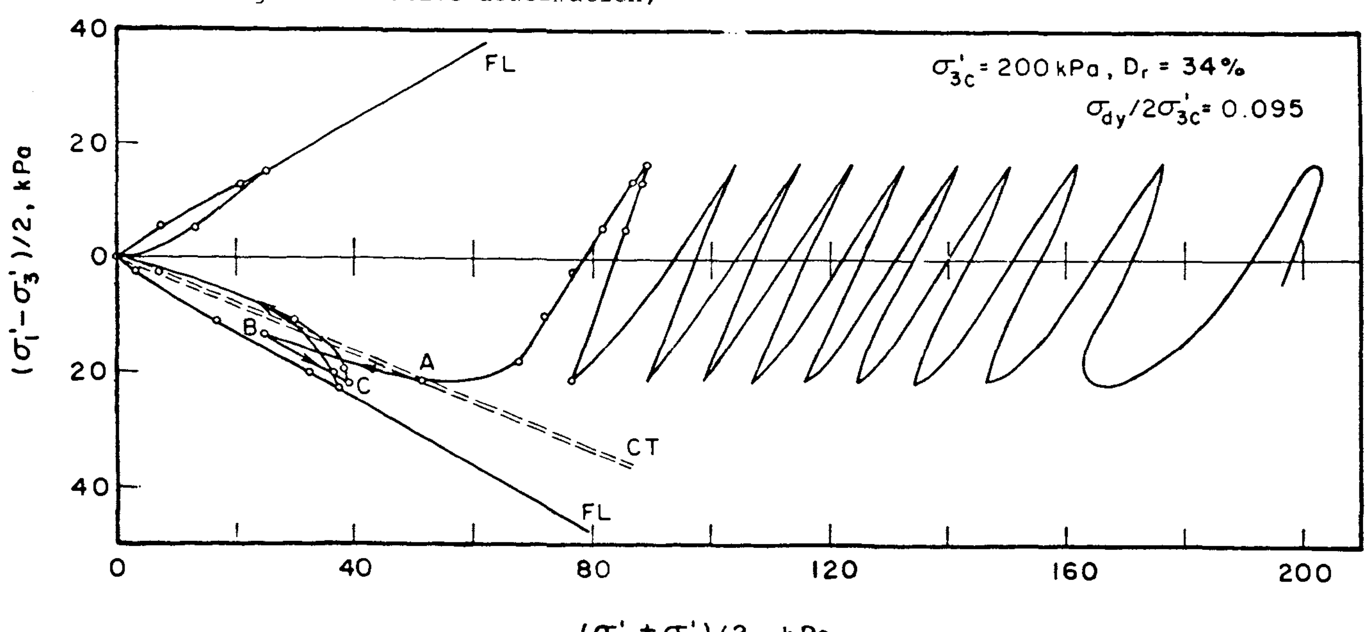

Fig. 1. Effective Stress Paths for Cyclic Loading Undrained Test on Isotropically Consolidated Loose Ottawa Sand.

Reliability of Experimental Data

a) Uniformity of Laboratory Samples

Most of our understanding of the fundamental mechanical behaviour of soils has come from laboratory tests under controlled conditions. The resistance of saturated granular material to cyclic loading has been one of the areas of such studies. From these studies it has been found that the relative density of sand is an important parameter controlling liquefaction resistance of sand. In many fundamental studies on liquefaction, sand samples are prepared by sedimentation through water. Higher relative densities are obtained by vibrating the sample during deposition. It is felt that extreme care is required to prepare a dense sample of uniform density throughout. In conventional procedures, densification of the sample to the desired relative density is generally carried out prior to levelling of the top surface and seating of the loading cap. A loose layer of sand tends to form at the top due partly to levelling action and partly to seating of the loading cap on the sand surface. Such a loose layer in an otherwise dense sample would lower the overall liquefaction resistance of the sand sample. In the improved procedure, the sand is deposited loose and is not densified until after the loading cap has been seated on the sand surface and a small seating load is maintained on the cap during vibration. The loading cap thus follows the settlement of the sand surface and assumes a proper seating, while the entire sample gets uniformly densified without development of loose zone at the top. It is believed that this manner of densifying the test samples results in the development of full liquefaction resistance of sand at the prepared average density. Such a conclusion is supported by extensive laboratory tests in which dramatic increase in resistance to liquefaction was noted if dense samples were prepared by the improved technique.

Fig. 2 shows data on the resistance to liquefaction of normally consolidated Ottawa sand as obtained in the simple shear apparatus.

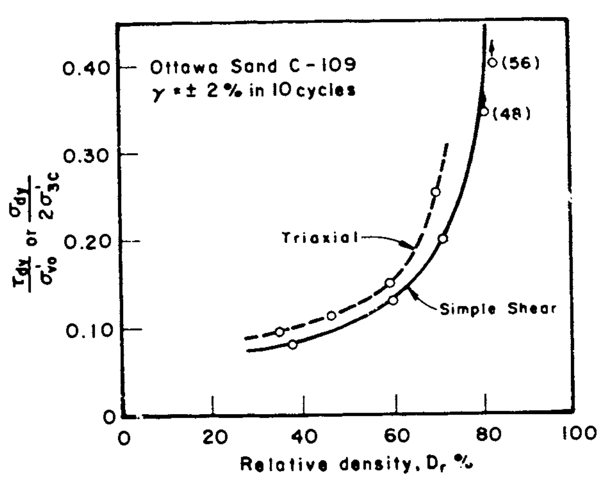

Fig. 2 Resistance to Liquefaction of Ottawa Sand Under Triaxial and Simple Shear Conditions

Improved sample preparation techniques described previously were used in this study. In figure 2, the cyclic stress ratio τ_{dy}/σ'_{vo} to cause ±2% shear strain in 10 cycles is shown as a function of relative density. It may be seen that the liquefaction resistance increases with increasing relative density and very marked so for relative densities in excess of about 70%. For relative densities in excess of about 80% it is almost impossible to develop ±2% shear strain in 10 cycles even under cyclic stress ratios in excess of 0.40. The numbers in parentheses in Fig. 2 represent the actual number of cycles (not 10 cycles) to develop ±2% strain in these dense samples. The vertical asymptotic nature of the liquefaction resistance curve indicates that liquefaction is unlikely to occur irrespective of the level of cyclic stress ratio if sand has a relative density in excess of about 80%. Such a conclusion seems apparent from the analysis of field records of liquefaction by Seed (1976) and Castro (1975). In the literature, however, sand has been characterised as prone to liquefaction regardless of its relative density. It is felt that such a conclusion has been drawn from laboratory studies in which the uniformity of samples, particularly, at higher relative density was not assured.

Fig. 2 also shows data on liquefaction resistance of the same sand as obtained in the triaxial apparatus. These results were also obtained by using similar careful experimental techniques as used for simple shear results. Again a vertical asymptotic nature of the liquefaction resistance curve may be noted corresponding to a relative density value of about 75%.

b) Accuracy of Measurement

Apparatus flexibility in some cases can contribute significantly to the process of measurement of soil deformations. If not properly accounted for, this can lead to erroneous data base for development of constitutive relations of soils. One example of such errors is presented below.

Martin et al (1975), in their discussion of fundamentals of liquefaction have presented a method of relating volume changes in drained cyclic loading tests to pore pressure changes in corresponding undrained tests. The equation relating these quantities at the completion of a loading cycle is

$$\Delta u = E_r \, \Delta \, \varepsilon_{vd}$$

in which Δu = increase in residual pore pressure for the cycle, $\Delta\varepsilon_{vd}$ = net volumetric strain increment corresponding to decrease in volume occuring if load cycle was applied under drained conditions and E_r = tangent modulus of the one dimensional unloading curve at a point corresponding to the initial vertical effective stress from where the cycle of loading was initiated. Martin et al used the NGI apparatus for determining one dimensional rebound characteristic of crystal silica sand reported in their study. The NGI apparatus, because of its use of a reinforced membrane, is a rather flexible apparatus and is not able to simulate accurately the condition of one dimensional strain during unloading. Consequently, considerable error can arise in the measurement of vertical effective stress σ'_v vs recoverable volumetric strain ε_{vr} relationship and hence the values of E_r, which are simply the slopes of this relationship. That such would be the case is illustrated in Fig. 3 by the comparative data on one dimensional rebound characteristics of crystal silica sand using a smooth ring consolidometer, representing true one dimensional unloading and the results obtained by the authors using the NGI device. It may be seen in Fig. 3 that the true one dimensional rebound modulus, E_r over most of the σ_v' range is about twice that measured using the NGI apparatus.

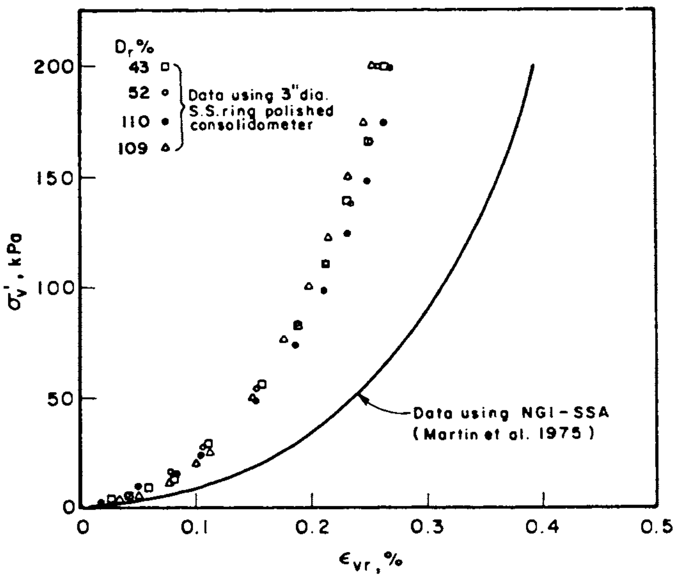

Fig. 3 Recoverable Strains During One Dimensional Rebound - Crystal Silical Sand.

Thus the porepressure model proposed by the authors would be substantially influenced by the technique of their measurement.

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Seed, H.B., (1976), "Evaluation of Soil Liquefaction Effects on Level Ground During Earthquakes," State-of-the-Art Paper, ASCE, National Convention, Philadelphia.

Discussion by Edward Prost on "Dilation Rate as a Measure of Liquefaction of Saturated Granular Material".

The authors outline a procedure similar to that outlined by Seed (1971) for prediction of liquefaction potential in sands. Here, however, a value termed the dilation angle, v, determined from pressuremeter tests is the correlating value.

Volume change characteristics have long been considered a primary factor for liquefaction potential. Casagrande has long believed that dense sands will not liquify due to their strongly dilative behavior upon shearing. Luong and Sidaner (1980) have postulated the "characteristic state" approach from which behavior of a soil can be predicted qualitatively for any type of shearing, static, or cyclic.

In this paper the authors conclude that for a given soil the dilation angle is dependent only upon relative density and confining pressure. These are two major factors affecting liquefaction potential, however, other factors of great importance exist. Of these are stress history and aging, which may lead to slight or heavy welding or cementation of grains. These factors would have little effect on the dilation angle determined at 10% strain. At the same time they might tremendously increase the stress required for liquefaction in the case of loose sands under low confining pressures.

The SPT has an advantage over the pressuremeter in this case since concretion and welding will be reflected by increased blow-counts due to expended energy to overcome these constraints. However, it is not certain as to whether blow-counts are increased in the same proportion as liquefaction resistance.

The effects of other factors such as grain size distribution and drainage of the deposit are left to engineering judgement in each simplified procedure and should never be neglected.

The use of the pressuremeter in field testing has two primary advantages: (1) It allows for accurate measure of fluid pressure rather than the subjectivity of energy input of the SPT. (2) The values of ϕ' and v correlate well with those obtained from triaxial tests on loose sands when derived using the methods described by Hughes, et. al. (1977).

Disadvantages to its use are added expense, present limited use, and sensitivity to disturbance primarily due to the critical distance of the cutting tool to the end of the tube which is a function of the individual soil properties. This last disadvantage could lead to the same type of user subjectivity experienced with the SPT. Another concern may be the difference in the method of shearing by the pressuremeter from that of triaxial and simple shear. Theoretical analysis should be made and tests should be performed on medium to dense sands in which differences would be most pronounced.

From Figure 3 of the original paper dilation angle is shown also to be a function of the particular soil. From this graph, an equal dilation angle in the two sands tested at the same confining pressure would correspond to differences in relative density of about 10%. This difference could greatly affect the liquefaction potential curve, especially at high relative density or dilation angle where resistance increases asymototically. It should be determined from further testing, the sensitivity of the dilation angle-relative density relationship to differences in angularity of soil grains, grain size, and gradation of the sand.

The liquefaction potential data, being derived from SPT results, is subject to any inherent errors in the test itself plus the subjectivity of Gibbs and Holtz (1957) relationship for relative density. Thus, although, it has been determined that the pressuremeter is a useful device for determination of liquefaction potential, further data, independent of these sources must be compiled to show merit of its use over the commonly used SPT.

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Discussion by Shobha K. Bhatia, Assistant Professor, Syracuse University, Syracuse, New York, on "Liquefaction of Soils"

Needless to say that the papers presented in this session (Liquefaction of Soils) were excellent and thought -provoking. In addition to the superb state of the art paper presented by Dr. W.D. Liam Finn, a total of twenty papers were presented. All these papers provided input to the ultimate goal of predicting the liquefaction potential in many ways diagrammed in Figure 1. If we superimpose Figure 2 on Figure 1, we get the statistical

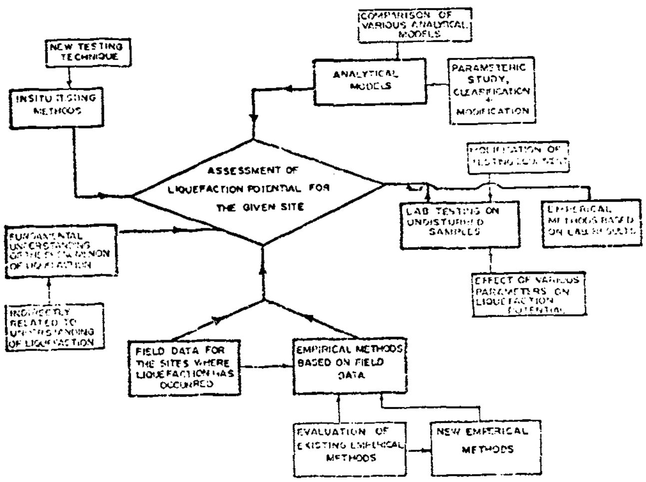

FIGURE 1 INPUT TO LIQUEFACTION ASSESSMENT FROM VARIOUS SUBCATEGORIES

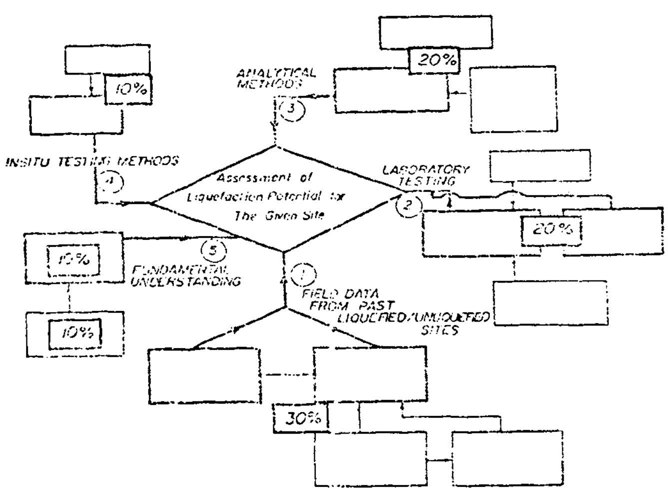

FIGURE 2 PERCENTAGE CONTRIBUTION OF PAPERS IN SESSION 2 TOWARDS THE ASSESSMENT OF LIQUEFACTION POTENTIAL.

breakdown of the papers in every subsection (given in boxes in Figure 1). Such a breakdown is not only critical for quantitative purposes but is also important to pinpoint the areas of thrust of the papers presented here.

The papers can be grouped into five general classes given in Table 1. The number of papers presented in each class served the basis of the statistical data presented in Figure 2. It is not possible for me to discuss all the papers here due to time constraints. Therefore, I have selected for discussion one paper from each of the five classes. Perhaps I should add that the papers chosen for discussion are not randomly selected. Several criteria such as (i) representation of the field (ii) solid contribution (iii) impact on future direction have gone into the selection.

The papers selected and their respective classes are as follows:

TABLE I

DISTRIBUTION OF PAPERS IN VARIOUS CATEGORIES FOR ASSESSMENT OF LIQUEFACTION OF SOIL

Categories	Sub-sections	Papers by	No. of Papers
Field data from past earthquake	Evaluation of Existing Emperical Relationships	Gil, Takimatsu & Yoshiaki, Talagahov, et. al.	6
	New Empirical Relationships.	Yegian & Vitelli, Davis & Berrill, Iwaski, et. al.	
Analytical Methods	New Models.	Oka & Murose, Sato, et. al.	3
	Modification or clearification of existing model	Martin, et. al.	
Laboratory Methods	Affect of various parameters on Liquefaction	Ishibashi, et. al. Champnella & Lim.	4
	Empirical approach	Vaid, et. al., Haldar	
Insitu Testing Methods	New Approach	Zhou, Forrest et. al.	2
Fundamental Understanding of Liquefaction	——	Zhong-qi, Wang Long et. al., Haldar, Roe	5

1. Field Data from Past Liquefied/Unliequefied Sites - Paper by Carrillo Gil, "Comparative study of Soil Liquefaction Potential during the 1970 Peru earthquake".

2. Laboratory Testing - Paper by Vaid, Byrne and Hughes, "Dilation angle and Liquefaction Potential".

3. Analytical Method - Paper by Luong and Sidaner, "Undrained Behaviour of Cohesionless Soil under cyclic and Transient Loading".

4. Insitu Testing Method - Paper by Zhou, "Influence of Fines on Evaluating Liquefaction of Sand by CPT".

5. Fundamental Understanding of Liquefaction - Paper by Wang, "Mechanism of Soil Liquefaction".

Carnillo-Gil reports the results of the analysis of the sandy soil along the coastal area of Peru to determine the liquefaction potential under a very intense earthquake. The results indicate that the specific Chimbote area (a part of a city) presents a dangerously unstable condition due to the high water table position and lower density of soil.

The two methods of analysis which went into this study are the ones proposed by Seed and Idriss (1971) and Schnabel et. al. (1972) and were proposed to evaluate dynamic shear stresses that cause the liquefaction of a specific soil. The analysis by Carrillo-Gil indicates that both methods yield similar results. And, to evaluate the liquefaction potential the calculated dynamic shear stresses were compared with the shear stress required to cause liquefaction for the specific soil. On the basis of his analysis of the soil, the paper points out that, in the case of another severe earthquake, similar to that of the 1970, the Chimbote area is likely to liquify unless the ground water table is lowered.

The paper by Vaid et. al. is interesting and significant in many respects. Most importantly, the paper claims that the liquefaction resistance can be expressed in terms of the dilation angle of soil. By comparing the dilation angle for soil measured in the field with the value of the dilation angle measured in laboratory, an assessment of the possibility of liquefaction can be obtained. The paper suggests that self boring pressuremeter can be used to evaluate the dilation angle of soil and in a laboratory the dilation angle can be obtained by performing simple shear or triaxial tests. Thus, from laboratory tests relationships between the cyclic shear stress ratio required to cause liquefaction and the dilation angle can also be obtained.

The authors also show that liquefaction resistance can be correlated with the relative density, the corrected dilation angle (corrected for vertical confining stress) or the corrected blow count, and a chart is presented in terms of three parameters for Ottawa sand. Their method is certainly interesting and deserves to be substantiated further with more field and laboratory data for other type of soils.

The paper by Luong and Sidaner is refreshing and introduces a new concept, "The Characteristic threshhold State" for cohesionless soil associated with interparticle friction angle. The characteristic state of cohesionless soil constitutes an important factor for the mechanical aspect of the behavior of soil and can be related to the interlocking capacity of the granular material. The anthors state that liquefaction of saturated soil can occur only under cyclic loading conditions until the "characteristic threshhold state" is achieved by soil. It is shown that in the case of undrained anisotrophic loading, the "characteristic threshhold" defines the average mobilized friction angle; effective stress path is stabilized on the C.L line. These claims are substantiated with the data drawn from SF sand.

In order to evaluate the liquefaction potential of sand, Zhou proposes an empirical equation which is based on static cone penetration results. This empirical equation is the result of field test data from the Tangshan earthquake area where the soil is primarily a clean sand with little fine contents. The proposed equation includes the terms of the epicentral distance, depth of water table, thickness of unliquified cohesive soil layer and the mean depth of sand layer concerned.

The equation enables one to abstract a relationship between critical cone resistance values and the intensity of earthquake. Although, the equation correctly predicted the liquefaction of sites of Haicheng earthquake, it over predicted the values for the critical CPT in the Lutai-earthquake area. The reason for such a deviance is attributed to the different soil characteristics, i.e. there were more fine contents than anticipated for the soil for which the equation was originally proposed. In short, the equation is of great value to predict liquefaction potential. Attempts should be made to extend its predictive power to account for soil with large fine contents.

The paper by Wang presents the physical meaning and mechanism of soil liquefaction. Liquefaction is viewed as the transformation of any substance into a liquid. For cohesionless soil, such a transformation is caused by seepage pressure, monotonous and cyclic loading or shearing. Those processes are explained on the basis of stress evolution. It is concluded in the paper that the state of stress in saturated cohesionless soil is bounded by a limit equilibrium condition, and ultimately, approaches "hydro-static pressure" during the process of liquefaction. Also, it is claimed that soil liquefaction can be correlated with the fabric characteristics and drainage of the soil mass. Needless to say, it is an interesting hypothesis which needs to be supported by further research. The paper also claims that the strain does not seem to be the proper basis for the definition of soil liquefaction.

From the above discussion, it is clear that the contribution of the papers is significant and that the area of liquefaction is especially benefitted by such research. The challenging job that remains for future studies is to examine questions such as:

(1) How can one derive an optimal benefit from their topic specific research to develop a unified theory of liquefaction?

(2) Even, more importantly, is it feasible to develop a unified theory of liquefaction to effectly utlize the insights of such topic-specific research?

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Discussion by Pedro A. DeAlba,
Assistant Professor, Civil Engineering,
University of New Hampshire on "Potential for Liquefaction Due to Construction Blasting" by J.H. Long, E.R. Ries and A.P. Michalopoulos.

The authors have made a very useful contribution by presenting this clearly documented case history. The topic of pore pressures generated by construction blasting is still poorly understood, and prediction techniques are in their infancy. This is recognized in the paper, where the authors point out that their prediction technique considerably underestimates the measured excess pore pressures induced by the test blasting; i.e. for test number 7, Table II, excess pore pressure measured at piezometer 1 (closest to the blast) is about 1048 psf (50 kPa) whereas the predicted value would be less than 400 psf or 19 kPa (discussor's calculation).

It is interesting to compare these values with empirical expressions such as that reported by Studer and Kok (1980):

$$\Delta U_{sd}/\sigma_0' = 1.53 + 0.77 \ln \frac{W^{1/3}}{R} \quad \text{(lower bound)}$$

$$\Delta U_{sd}/\sigma_0' = 2.15 + 0.74 \ln \frac{W^{1/3}}{R} \quad \text{(upper bound)}$$

Where ΔU_{sd} is the blast-induced residual excess pore pressure, σ_0', is the initial effective stress, W is the explosive weight in kg_f and R is the source distance in m.

For test no. 7, the predicted pore pressure ratio, $\Delta U_{sd}/\sigma_0'$, would be between 0.5 and 1.0. Consequently, the predicted excess pore pressure would be greater than 2800 Psf (134 kPa). The expression reported by Charlie et. al. (1979) for radius, R_{max}, of the liquefied zone might also be used:

$$R_{max} = K_3 W^{1/3} \text{ (m)}$$

Where the empirical constant K_3 = 5 in this case. Again, for test number 7, the predicted value of R_{max} is somewhat in excess of the piezometer/source distance, and liquefaction would be predicted. In making these comparisons, it should be noted that excess pore water pressures were measured in open-standpipe type piezometers and that the maximum response for the piezometer shown occurred about seven min. after the blast. The discussor would suggest that, if these long response times are typical of the reported values, then these values are probably lower than the peak residual excess pore pressures actually induced by the blasting. It is very likely that, during the time required for water to flow into the piezometers, significant pressure dissipation occurred in the source deposit. Thus the values predicted by Studer and Kok may be closer to reality.

It is the discussor's opinion that the proposed method, while undoubtely more attractive than purely empirical expressions, has two important sources of uncertainty:

(a) The asssumption that the longitudinal strain calculated from particle velocity considerations is equal to the longitudinal strain in cyclic triaxial and torsional shear tests. For the case history presented, it is obvious that the predicted strains, and consequently, the predicted pore pressures are too low.

(b) The stress ratio versus pore pressure ratio relationship will be affected by the sample reconstitution technique used in the laboratory (Mulilis et. al., 1975)

It might further be suggested that the tests on which the method is based are essentially cyclic shear stress tests; Charlie et. al., (1980) have pointed out the importance of considering the contribution of the compression wave to pore pressure buildup. It is the discussor's opinion that, in the near field of a blast, both shear and compression waves contribute to the resulting volume change tendency of saturated sands and consequent pore pressure increase. To the discussor's knowledge liquefaction have not been considered.

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Discussion by Wang Zhong-qi, Deputy Chief Engineer, Academy of Building Research of China on "Analysis for Liquefaction: Empirical Approach" by M.K. Yegian and B.M. Vitelli.

The paper presented by Mr.Yegian et al provides an improved analytical method to evaluate soil liquefaction potential. With an expanded list of case histories as background, the authors proposed earlier the Liquefaction Potential Index (LPI) and the Coefficient of Variation of LPI (V_{LPI}). Such an empirical approach will merit due appreciation.

In relating soil liquefaction case histories for predicting liquefaction potential, I did ever hesitate about a logical consideration:

Whether it is valid to measure soil parameters after an earthquake (including SPT data) on a liquefied site where no given data available beforehand and one have to correlate them with liquefaction behaviour and in turn apply such correlation for prediction. The question arises as whether soil parameters (e.g. relative density of sand) collected after an event can represent those of the natural soil deposit before that event. In order to make further evidence on this uncertain problem, electrical static cone penetration tests were performed both before and after Tangshan earthquake 1976 (1). It is shown by contrast that for recently deposited silty sand and fine sand layers in level ground, in relative density once occurred during earthquake shock will tend to recover after the shock. And after one year or so, it will become as it was under the same overlying pressure, even though the thickness of the liquefied sand layer may reduce. If such an evidence reflects a general rule in a broader sense, the question will be answered. It is hoped that more practical observations in various sites will be beneficial as to make further confirmation on such problem.

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Discussion by Pedro A. DeAlba, Assistant Professor, Civil Engineering, University of New Hampshire on "Assessment of Liquefaction Potential Based on Seismic Energy Dissipation", by R.O. Davis and J.B. Berrill

The authors present a simple and attractive argument in favor of directly relating liquefaction potential to the work exerted by the earthquake on the problem material as measured by the product of the arriving energy density times a site dissipation function.

In the proposed method, the site conditions are characterized by an SPT value normalized to a standard effective overburden stress and the earthquake by magnitude and epicentral distance.

Site characterization through a normalized SPT value, which is considered to reflect the density and stress history characteristics of the sand, is also the basis of the semi-empirical method proposed by Seed (1976) and is widely accepted in practice.

It is the discussor's opinion, however, that the proposed calculation of incident earthquake energy by assuming isotropic energy radiation and ignoring path-dependent attenuation effects does not fit observed earthquake behavior, and considerably affects the general applicability of the method.

To illustrate this point the following liquefaction potential calculation based on the simplified method described by Seed (1976) is proposed.

Site conditions: Sand with groundwater level at a depth of 5 ft (1.5m), normalized blowcount, N_1, of 11 bpf (corresponding to a relative density D_r = 54% in normally-consolidated, recently-deposited sand). Consider liquefaction potential at a depth of 15 ft (4.6m), with initial effective stress σ_o' = 0.59 tsf = 56 kPa.

Stresses and acceleration required for liquefaction: stress ratio, τ/σ_o' = 0.10 for earthquake magnitude M = 8.25; τ/σ_o' = 0.16 for magnitude M = 6.0. The differences in required cyclic stress levels are due to the different numbers of cycles of motion typical of the two events considered.

From these stress ratios, and for the site conditions considered, it is possible to back-calculate the approximate maximum ground surface acceleration levels required to produce the required cyclic stresses: for M = 8.25, a = 0.105g; for M = 6.0, a = 0.167g.

From empirical correlations between earthquake magnitude, peak ground acceleration and epicentral distance for the west coast of North America (i.e. Housner, 1965; Page et al. 1972), the distances at which such events might occur in order to produce liquefaction at this site may be calculated. The values proposed by different researchers vary somewhat but the ratio of epicential distances, r(M=8.25)/r(M=6.0) remains on the order of 3 to 4.

From the authors' eqs (3) or (11), however, for the earthquakes considered it is found that the epicential distance ratio r (M=8.25)/r(M=6.0) is about 49 to produce the same energy density at the site. Thus, by the proposed method, a magnitude 8.25 earthquake would produce liquefaction for these site conditions at an epicentral distance 49 times greater than the magnitude 6 event. This is not in agreement with earthquake behavior.

It may therefore be concluded that path-dependent attenuation of earthquake energy is critical to correct liquefaction prediction; this effect must be accounted for in the proposed method before it can be considered a practical tool.

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Discussion by Bruce J. Douglas,
ERTEC Western, Inc., Long Beach, CA
on "Liquefaction of Soils"

The use of insitu test methods for assessment of liquefaction potential of cohesionless soils is receiving increasing attention from practicing engineers. The traditional method of assessing liquefaction potential using in situ test measurements has been through relative density correlations. Such correlations have been used, for example, with Standard Penetration Tests (SPT), Cone Penetrometer Tests (CPT) and, more recently, Pressure Meter Tests (PMT). For the case of the SPT, direct correlations also now exist between liquefaction potential and SPT blowcounts. Use of these correlations recognizes that other factors besides relative density affect the liquefaction potential of sand.

Unfortunately, all of the existing in situ test methods have difficulties when applied to evaluation of liquefaction potential of silty soils. Relative density determinations do not apply to such materials, and both the liquefaction potential (high cyclic strain potential) and the insitu penetration resistance are affected. As it is now recognized that silty, low cohesion soils with PI values up to about 10 are potentially liquefiable, it is clear that more research is needed on this subject. This has been recognized in several papers presented at this conference.

Recent investigations performed by Ertec using the quasi-static electric cone penetrometer in silty soils have provided interesting data regarding liquefaction potential of such soils. Soils that develop high pore pressures during continuous cone penetration typically have very low side frictions. These soils, which plot in Zone 1 of the electric CPT-Soil Behavior Type classification chart shown in Figure 1, have been found to comprise silty sands, sandy silts, and silty or sandy clays with PI values up to 11. Typically, the soils have 20 to 30 percent fine sand content with less than 15 percent clay sizes. The rest of the material is silt-sized. In addition, the materials of Zone 1 typically have Liquidity Indices (LI) close to 1.0.

As part of the above study, samples of these Zone 1 materials, as well as clean sands, were subjected to cyclic triaxial and cyclic simple shear tests. The results of these tests are summarized in Figure 2, where it can be seen that the cyclic strength of the clayey soils lying in Zone 1 (such as point 1), are as low as the clean sand strengths. Higher cyclic strengths (such as point 2) were obtained for similarly graded clayey soils of lower LI value, which were found in Zone 2 of Figure 1.

In summary, then, it appears the CPT measurements can be used to distinguish a range of soil types (including silty or clayey soils) susceptible to liquefaction. At this time, quantitative assessments of liquefaction strength for Zone 1 materials from CPT data, as attempted in several papers submitted to this conference, have not been performed by Ertec. However, it appears that such assessments should include the effects of in situ pore pressure generation, as indicated by either Friction Ratio or actual pore pressure measurements, as well as grain size and density characteristics.

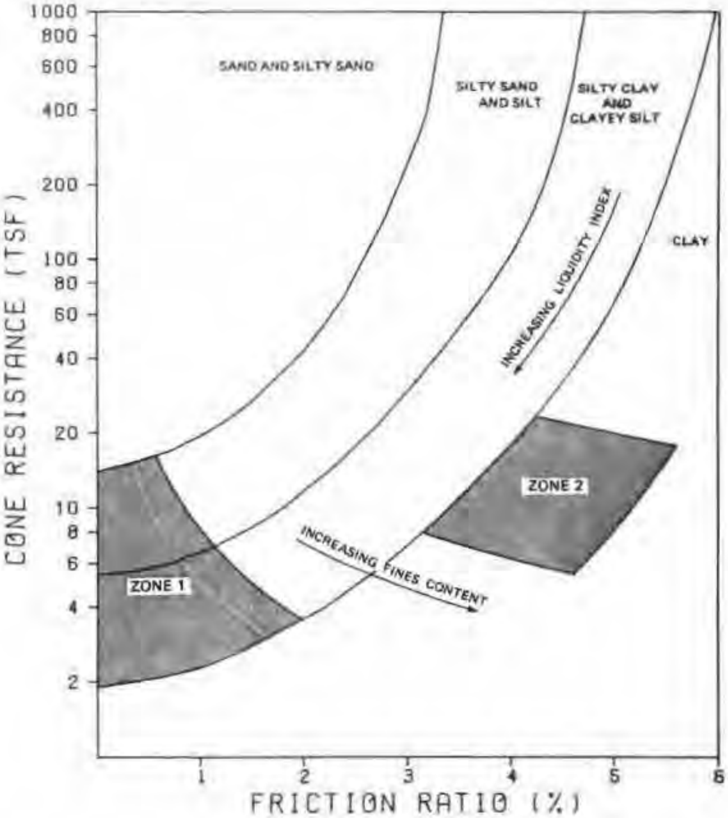
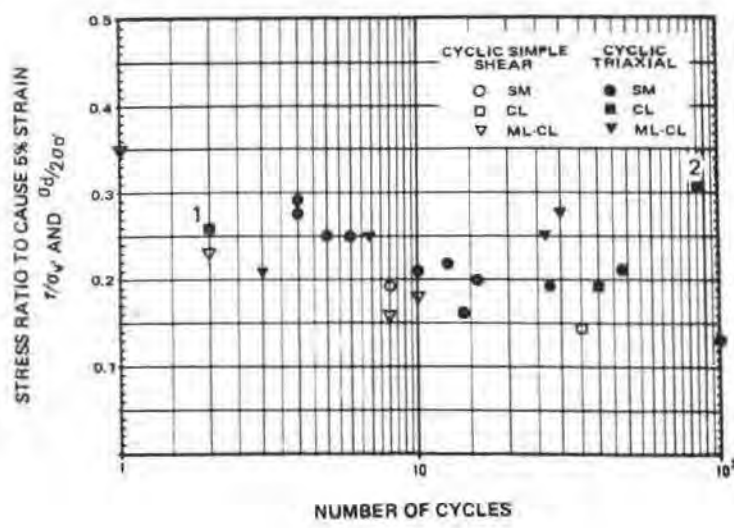


FIGURE 1

ZONES OF CPT SOIL BEHAVIOR TYPE CLASSIFICATION CHART



AUTHOR'S REPLIES

Closure by James B. Forrest, John M. Ferritto, and George Wu.

The authors thank Mr. Zhong-qi Wang for his valuable comments regarding the liquefaction potential of silty sands. As noted by many workers, particularly in Japan, the presence of fines markedly reduces the penetration resistance of sands without a comparable increase in liquefaction potential. Although the grain size distributions for the finer fractions of soil have been omitted in the paper, reexamination of laboratory results shows total fines contents of up to 18% for the critical soil zones (see Figure 10), with a large portion of these in the clay size range. Inserting typical values for the clay and silt percentages into Mr. Wang's Equations 1 or 2, one observes liquefaction potentials near the range of borderline liquefaction failure. Thus, the cone penetration predictions of liquefaction are in reasonable agreement with the cyclic triaxial test data.

Closure by Zhong-qi Wang.

The author wishes to express his appreciation and agreement to the discussor Mr. DeAlba's comment. The only thing to be added is that the effect of reliquefaction on the density of the liquefied layer itself is still controversial and contradictory from both theoretical and practical points of view. It is well known theoretically that dissipation of excess pore pressure will result in densifying the liquefied material. However, practically either by site investigation or laboratory testing, the liquefied layer or soil sample tends to be weakened or even loosened immediately after liquefaction. By the author's opinion, this general manifestation might be in most cases due to sudden collapse of soil skeleton during liquefaction and the soil particles will rearrange during pore water dissipation which has not been considered to be associated with the liquefaction mechanism so far. This is what we are searching for necessary improvement both in theory and engineering practice.

Closure by A. Carrillo-Gil.

The establishment of an unified theory to determine the liquefaction potential of a specific soil, must consider principally the practical engineering application of a method, based as well in the theoretical contributions derived from the knowledgement of the phenomenon, through the empirical results obtained from the field data, as in the observations of the soil behaviour combined with the practice of simple field and laboratory tests, but, in any case with extremely theoretical or sophisticated speculations which will require complicate tests that do not lead commonly to the obtaining of a real model of the soil behaviour and which, in most cases represent difficult methods to be applied to the common problems due to its incoherence and difficult interpretation.

In the paper presented,we reach to the conclusion that, the methods based in field observations are well adjusted with a more elaborated theory, and thus, we conclude thinking that this must be the way to develope an unified liquefaction theory in order to approach the phenomenon prediction to the common practice of the profession.

AUTHOR'S REPLY

Closure by M. P. Luong to discussion by Y.P. VAID

The characteristic threshold η_c appears to be independant of initial sand density, degree of anisotropy, applied stress path in the (p,q) diagram and thereby of initial consolidation stress ratio and cyclic stress level.

It divides the permissible stress space into two regions :
(1) subcharacteristic region corresponding to an interlocking of grain structure or contractancy;
(2) surcharacteristic region where disaggregation of granular material or dilantacy occurs.

Thus a closed load cycle in the subcharacteristic domain exhibits a contracting soil behaviour illustrated by an irreversible volume contraction (or an irreversible increase of pore water pressure) whereas a closed load cycle in the surcharacteristic domain leads to an irreversible volume dilation (or an irreversible decrease of pore water pressure).

A very accurate experimental determination of the characteristic threshold η_c is readily available under either drained or undrained conditions : η_c is revealed by the appearance of a dilatancy loop (volume change or pore water pressure) during a load cycle crossing the characteristic line C.L.

Test results of an isotropically consolidated Ottawa sand reported by the discusser (fig.1) seem to show that the effective stress point A is not lying on the characteristic line because it represents the triggering of the grain structure collapse of loose sand. No volume dilation occurs after this stress state. Along the segment AB, sand volume is contracting and pore water pressure is increasing.

The stress path reaches the characteristic threshold defined by the inversion of pore pressure generation rate $\dot{u}$ at stress point B. Then the stress path climbs into the surcharacteristic region bounded by the characteristic line CL and the failure line FL. After unloading along the stress path CO, the effective stress path shows a anticlockwise dilatancy loop. Thus it can be claimed that Ottawa sand is also consistent with and strengthens the concept of a characteristic state.

The author thinks that relative density D_r useful for monotonic loading test is not a significant parameter for the understanding of the fundamental mechanical behaviour of cohesionless soils under cyclic loading.

In fact, densification of dense sands may be obtained easily by cyclic loading at large amplitude exceeding both triaxial compression and extension characteristic thresholds. The high amplitude loading benefits in a partial loss of strain hardening during the dilating phase in the surcharacteristic domain which breaks down the granular interlocking assembly. On each reload, the tightening mechanism induces new irreversible volumetric strains and recurs with a renewed material becoming each time denser.

Several tests results (Luong 1980) under constant confining pressure, constant mean stress p and constant circular stress in (p,q) diagram show a rapid stiffening of sand under cycles of alternating deviatoric stresses on both sides of q = o. The densification process of dense sands is associated with a dilatancy loop at compression and extension characteristic stress levels. The intermediate part corresponds to an irreversible tightening between two sequences of granular assembly reinterlocking which fills up progressively the existing voids.

This experimental result is in agreement with direct shear tests carried out by Youd (1972) on Ottawa sand : each shear cycle formed a similar sequence of contractancy-dilatancy while an irreversible volumetric strain accumulated during cycles, reaching the relative density of $D_r \simeq 128\%$ (ASTM norm D 3049-69) at the end of 10,000 shear cycles having an amplitude of ± 0.51 mm.

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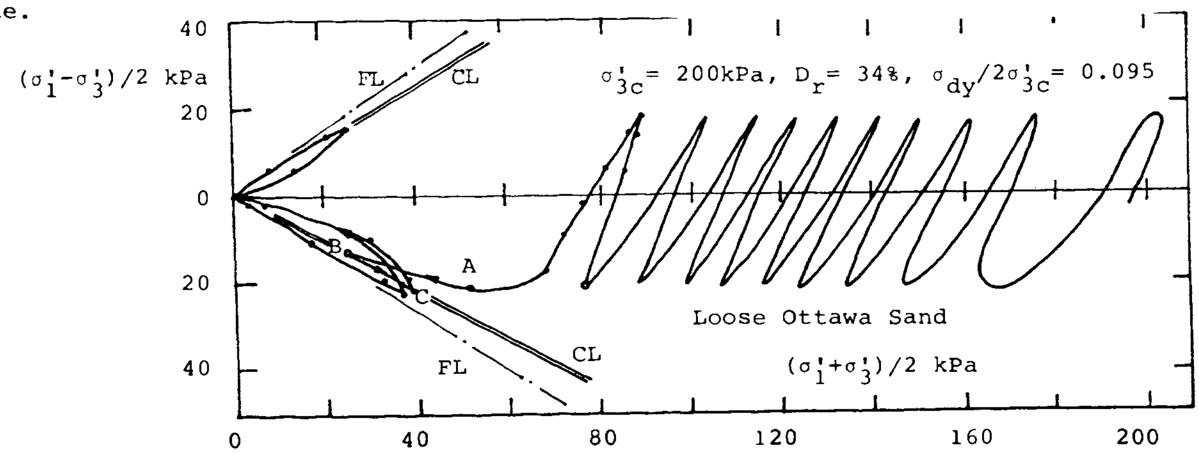

Fig.1' - Correct interpretation of the Characteristic State Concept for a loose sand.

AUTHOR'S REPLY

Closure by M. P. Luong to discussion by Ed. PROST

The use of SPT as well as pressuremeter in field testing allows the determination of useful in-situ values of global mechanical properties leading to good correlation factors. However obtained results are generally sensitive to the homogeneity, the degree of anisotropy, the stress history, and so on ... of the soil mass.

The characteristic state concept for cohesionless soils offers a rather convenient framework for interpreting different cyclic aspects of granular soil behaviour :

. Under undrained conditions :

(1) sand liquefaction occurs only when load is cycled alternately on both sides of zero deviatoric stress and has reached the characteristic levels. The characteristic friction angle φ_c represents the average mobilized angle of interparticle friction.

(2) cyclic non-alternated deviatoric stress tests show a progressive tendency of the stress state moving toward the characteristic level and stabilizing there, i.e. cyclic softening is occuring.

(3) cyclic hardenning of sandy soils may be observed when undrained loads are cycled in the surcharacteristic region bounded by the failure line FL and the characteristic line CL. It leads to a stabilization of the granular material on the characteristic threshold. Irreversible strains accumulated during undrained loadings depend on the stress amplitude of cycles.

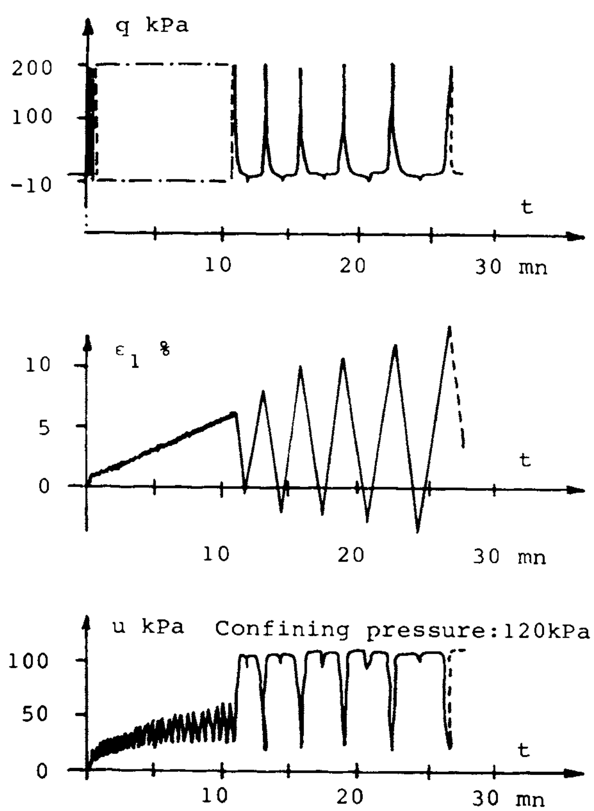

Fig.1 - Liquefaction of a Fontainebleau sand (e=0.720 ; Dr=62%).

. Under drained conditions :

(4) adaptation may be considered as obtained after a finite number of cyclic hydrostatic loadings.

(5) accommodation appears under radial or conventional loadings at a stress level $\eta = q/p$ smaller than the characteristic threshold η_c . Stress-volume change curves of sandy soils exhibit a clockwise hysteresis loop after unloading and reloading. This hysteresis susceptibility becomes negligible when the number of cycles increases.

(6) for η greater than η_c, the hysteresis loop disappears and cyclic loadings cause ratcheting behaviour. The soil volume increases and reflects the phenomenon of dilatancy of the grain structure. After unloading, a dilatancy loop is seen in an anticlockwise direction on any diagram where volume change is plotted. The dilatancy loop is a very practical and useful criterion for the detection of the characteristic threshold.

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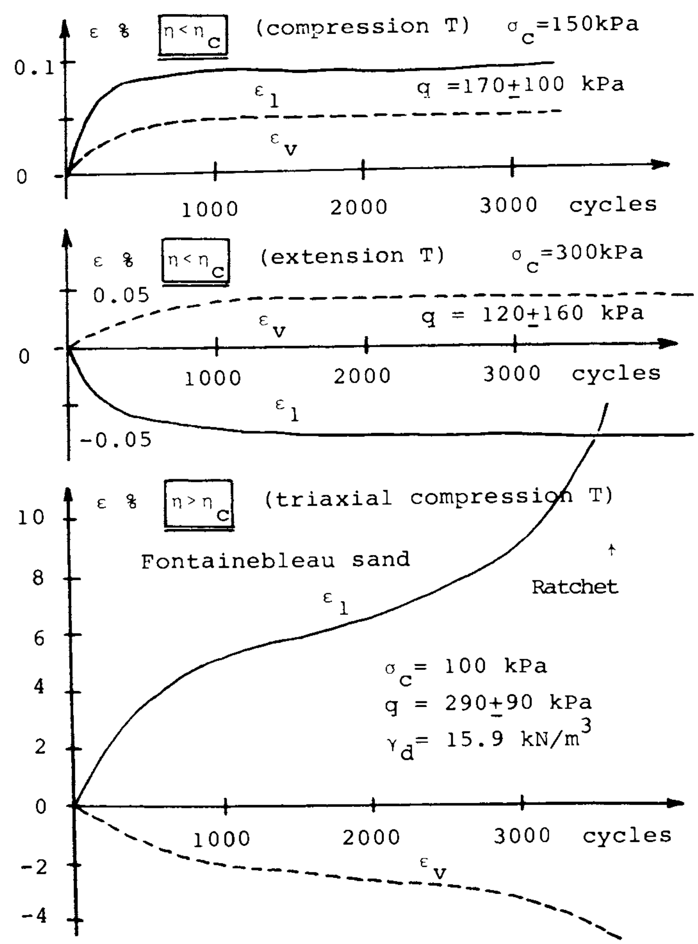


Fig.2 - Drained shearing tests under constant confining pressure.

AUTHOR'S REPLY

Closure by M. P. Luong to discussion by S.K. BHATIA

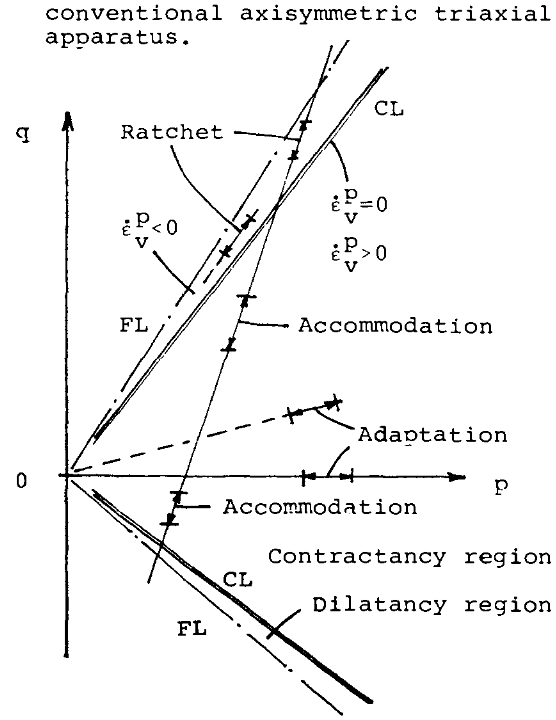
The aim of the characteristic state concept is to :
(1) grasp the fundamental aspects of the stress-strain behaviour of granular materials under cyclic and transient loading,
(2) consolidate the experimental data in order to define a characteristic stress domain where the resultant effect of load cycling is contractancy,
(3) suggest development of parameters for use in analytical and numerical models, and
(4) guide and interpret reduced model tests.

Extensive laboratory tests using the axisymmetric triaxial apparatus on various sands : Fontainebleau sand, Loire sand, carbonate Channel sand (Luong 1980, 1981), carbonate marine sediments (Nauroy et Le Tirant 1981) and Hostun sand (Thanapoulos 1980) substantiate the different rheological properties claimed by the characteristic state concept :

. The essential parameter for studying the mechanical behaviour of cohesionless soils is the generation of volumetric strains during loading stages. The friction angle φ_c is an intrinsic factor characterizing the interlocking capacity of grain structure in drained tests and the average mobilized friction angle under undrained conditions.

. The characteristic concept is explained and quite simply formulated on the basis of ordinary laboratory loading paths in the (p,q) plane. It can be defined by the existence of a dilatancy loop after unloading if the characteristic threshold is reached.

Fig.1- Diverse cyclic behaviours of cohesionless soils readily obtained from the conventional axisymmetric triaxial apparatus.



Dry sand or under drained condition

Under either undrained or constant volume conditions, the subcharacteristic region includes all possible effective loading points. As soon as this point reaches and crosses the characteristic line, it tends to parallel the CL line in the surcharacteristic domain. The length of the section followed determines the degree of memory-loss of preceding loading history, reloading being related to a new initial state.

. This concept becomes all important in the domain of cyclic loadings, facilitating the definition of a region of contracting behaviour for granular soils.

. A quite simple criterion of liquefiability is evident : the effective loading point reaches the origin of the (p,q) diagram (liquid behaviour for saturated sand) only for cases of alternated deviatoric stress loading on both sides of q = o .

. The salient features of granular soil behaviour under cyclic loading studied utilizing the conventional triaxial apparatus are easily interpreted within the framework of the characteristic state.

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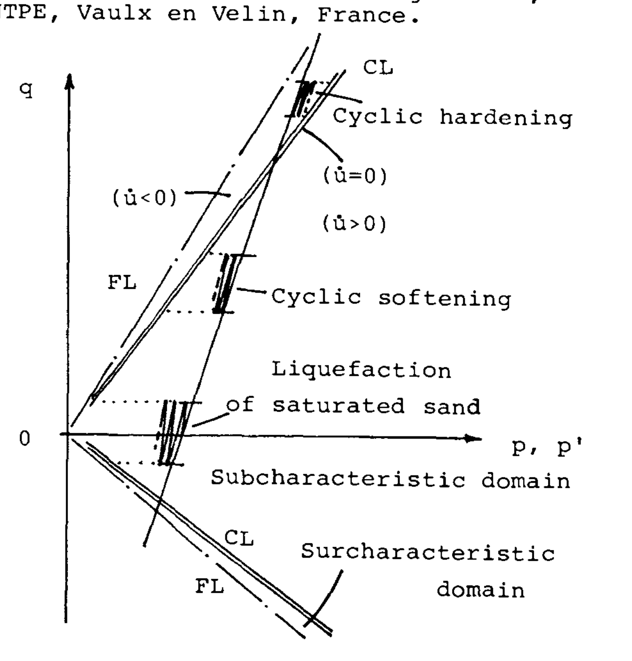

Sand under undrained condition

AUTHOR'S REPLY

Closure by S.G. Zhou

1. The author's emperical formula for evaluating the liquefaction potential of sand by CPT has been further confirmed by the tests in the following earthquake areas:

(I) Bohai earthquake (June 18, 1969; M=7.4; epicentre, east longitude 119°42', north latitude 38°12'; depth of seismic focus 35 km).

(II) Xingtai earthquake (March 8, 1966, M=6.8; epicentre, 114°55'E, 37°21'N; March 22, 1966; M=7.2; epicentre, 115°03'E, 37°21'N; depth of seismic focus 25 km).

(III) Yangjiang earthquake (June 26, 1969; M=6.4; epicentre, 111°45'E, 21°45'N; depth of seismic focus about 5 km).

The results of above-mentioned tests are concluded and listed in the table.

Intensity of earthquake	VII		VIII		IX	X
Situation of the sand	Liquefied	Unliquefied	Liquefied	Unliquefied	Liquefied	Liquefied
Number of tests	18	10	16	3	14	16
Failed in evaluation	1	2	0	2	0	0

It is shown that the author's emperical formula is suitable for evaluating the liquefaction potential of clean sand. The four tests, which were failed in evaluation, were carried out in the unliquefied districts of which two locations are silty sand with high content of fines. Therefore, the author's method should not be used for such silty sand without correction.

2. During Tangshan earthquake, severe liquefaction phenomena were also appeared in Tianjin area, and most of which were occured in the silty sand layer. The Third Railway Design Institute has carried out a lot of CPT and laboratory soil tests. It was also shown that the author's method used in silty sand should be corrected in accordance with the amount of fines. They have suggested an modified method for Tianjin area in which the corrections were suggested basing the ratio of the amount of clay particles (<0.005 mm) to the silt particles (0.05-0.005 mm).

3. Up to now, the case records in the silty sand are still not very much, and further field and laboratory tests should be carried on.

AUTHOR'S REPLIES

Closure by Yoshiaki Yoshimi.

The discusser essentially agrees with our paper and goes on discussing the SPT on a very broad basis which is outside the scope of our paper. After deliberations Tokimatsu and I have decided that we would not be able to prepare a meaningful reply to the discussion.

Closure by Fusao Oka.

I would like to reply to comments by discusser Pedro A. De Alba briefly. Generally, it is difficult to accurately determine the soil parameters of undisturbed soil samples, required to complete the proposed constitutive equations. The samples are always suffered some disturbance due to sampling technique and testing method. At least, it is necessary that the soil samples have to be put back into the insitu original stress state. But, it is basically possible to determine the parameters M_f^*, M_m^*, κ, λ and G' from the triaxial test results.

M_f^* and M_m^* and G' are considered to be a function of relative density. G' is also a function of O.C.R.(see Oka&Washizu 1981). The consolidation parameter κ has a great effect on the pore water pressure build-up during the earthquakes. The decrease in κ causes the increase in excess pore water . The variation of the value of permeability coefficient k also influences the liquefaction of the layers near the surface due to the upward seepage flow. The more general parametric study will be published in the future.

The proposed liquefaction model can predict the dissipation of pore water pressure after the earthquakes has stopped. The dissipation of pore water pressure is estimated by introducing the Darcy's type interaction between the pore water and soil skelton. The authors agree with the results by Seed et al.(1975). The calculation was not carried out beyond 10sec or 16 sec, because the time reqired for computaion becomes too large. But, the layers near the surface may be liquefied after an earthquake due to upward flow as the pore water pressure dissipates.

The post failure stress-strain relationship is introduced as a restiction for numerical calculation. The effect of limiting strain potential (or cyclic mobility) can be described by the proposed elasto-plastic constitutive model. If we attempt to include these effects in the analysis, the new assumption about the stress path after failure has to be introduced.

The authors accurately assume that the horizontal deformation gradient(strain) is zero(see Eq.(19)), but does not assume that the particle velocity in the horizontal direction is zero. Therefore, this assumption is compatible with the horizontal input shear motion at assumed at the base. To remove a cause of misunderstanding, the assumption has to be called "horizontal deformation gradient confined condition".

Closure by M. K. Yegian and B. M. Vitelli.

Mr. Z.Q. Wang's interest in our paper presenting an empirical approach to liquefaction analysis is appreciated. In answer to Mr. Wang's comments regarding Standard Penetration Test data, reference is made to the results of investigations by Koizumi (1964). Koizumi demonstrated that SPT values in sand change as a result of earthquake-induced excitation. He introduced the concept of critical SPT value, later used to define the extent of the liquefied sand layer at various sites in Japan.

The majority of SPT values for the case histories used in our investigations corresponded to data collected prior to earthquake shaking. Variability and uncertainties in SPT data due to the test procedure itself, as well as to earthquake occurrence prior to or following the collection of data, are recognized and acknowledged. It is in part for this reason that we have recommended that liquefaction analysis be made in a probabilistic manner, accounting for the various uncertainties present in both the analytical procedure and the parameters used.

Closure by R.O. Davis and J. B. Berrill.

We wish to thank Dr. De Alba for his interest in our paper, but to disagree with his conclusion. De Alba bases his argument on an examination of distances at which liquefaction would occur in an earthquake of a given magnitude. In particular, he examines the ratio between maximum distances at which liquefaction is predicted by our model for earthquakes of M = 8.25 and M = 6.0 respectively, and states that the value of 49 given by the model for this ratio does not agree with observed earthquake behaviour. We cannot accept this statement. On the contrary, this prediction agrees remarkably well with observed data. For example, the ratio 49 is very close to the value of 54 predicted by the expression:

$$\log_{10} r_{max} = 0.77\,M - 0.60 \qquad (1)$$

found by Kuribayashi and Tatsuoka (1975) from a large set of Japanese liquefaction data. (Here, r_{max} is the epicentral distance to the farthest point of liquefaction for magnitude M).

Finally, material attenuation is not completely neglected in our model. Its average effect is reflected in the value of the constant denominator of the function, γ. Frequency-dependent material attenuation has been included in a subsequent model not yet published, and while it does improve the fit of the model to historical data, its effect is by no means critical.

Reference:

Kuribayashi, E. and Tatsuoka, F. (1975), "Brief Review of Liquefaction During Earthquakes in Japan", Soils and Foundations, (15)4, 81-92.

Moderator's Answers to Discussions.

I agree with Roe, DeAlba and Celikkol that both shear and compression wave velocities have a role to play in liquefaction studies, for the reasons given in the Moderator's report.

The preliminary data presented in his discussion by Douglas, on identification of soil types by CPT, look promising, both for liquefaction and for general geotechnical engineering purposes. As noted by him, more research is needed on the relation between liquefaction resistance and the location of the soil in the Cone Resistance-Friction Ratio plot.

AUTHOR's REPLY

Closure by Pedro DeAlba

We are grateful to the moderator for classifying our report to this conference among new field methods. The compression-wave work presented is, however, a first-stage in the development of a field method for liquefaction-potential evaluation involving both shear and compression waves. We are currently completing our shear-wave work on Dover 40-50 sand. The results show that the most sensitive indicators of liquefaction potential involve both shear and compression wave propagation characteristics; in particular the ratio of compression wave velocity to shear wave velocity seems very promising.